WISCONSIN PLACE RESIDENTIAL

TECHNICAL ASSIGNMENT 2

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Introduction

Wisconsin Place Residential consists of 15 above stories and 2 below grade stories. The building is approximately 479,000 SF, stretching from 25 feet below grade to 142 feet above grade. The building consists of 432 units spread out over the 15 floors. The 13th floor contains a 1,000 SF pool for all tenants of the building. The two levels below grade are set aside for residential parking and are integrated with the parking for the mixed use development.

This report introduces a comparative analysis of four different alternative floor framing systems for Wisconsin Place Residential. The typical existing structure of the building is a two-way Post-Tensioned Flat Plate with normal weight concrete. A description of the system over a typical bay is contained in the following section of this report.

To remain consistent with typical design practices of residential construction, alternate floor systems were analyzed for the result of achieving a smaller floor sandwich dimension. These systems include:

- Redesigned Two-Way Post-Tensioned Flat Plate w/ Lightweight Concrete
- Precast Girder-Slab
- Two-Way Flat Plate w/ Normal and Lightweight Concrete
- Composite Deck w/ Non-Composite Steel Framing

Alternative systems were analyzed using loadings following ASCE 7-05. Due to the irregularity of the building, existing span conditions were modified for the alternate systems to remain consistent with typical and economical design practices. All analysis for alternative floor systems can be found throughout the Appendix. Each section contains a typical bay or frame as well as the summary of the analysis. Advantages and disadvantages of the alternative floor systems are described throughout this report. The conclusion contains a table which includes the overall depth, constructability, cost of the system, potential vibration problems, lead time, fire proofing and more.

Gravity Loads

The gravity and lateral loads were determined in accordance with ASCE 7-05. Live Loads were established using section 4 of ASCE 7-05. General assumptions for dead loads were made based on unit weights from ASCE 7-05. Instead of calculating every column and wall, I assumed an addition 10 PSF load on each floor.

Dead Loads:

<u>Construction Dead Loads:</u>

Concrete	150 PCF

<u>Superimposed Dead Loads:</u>

Partitions	20PSF
Finishes & Miscellaneous	5 PSF
MEP	10 PSF
Columns & Walls	10 PSF

Live Loads:

Floors	40 PSF
Canopy	75 PSF
Slab-On-Grade	100 PSF
Storage	125 PSF
Public Rooms and Corridors	100 PSF

Balconies	100 PSF
Lobby, Corridors, Stairs and Pool Areas	100 PSF
Penthouse, Mechanical Room	150 PSF
Elevator Machine Room	125 PSF
Roof	30 PSF
Roof Snow Load	27 PSF

Deflection Criteria

Maximum deflection of studs in exterior walls subject to wind shall be L/600 when used as a backup for masonry. For other materials, maximum deflection shall be L/360. Floor deck deflection shall not exceed L/360 under full live and superimposed loads. Dead load and a 20 PSF construction live load shall not exceed L/180.

Executive Summary

After completing the analysis of the four alternate systems, each system had unique advantages and disadvantages. The first alternate system proposed was a redesign of the existing Two-Way Post-Tensioned Flat Plate with Lightweight Concrete. Though lightweight concrete is more expensive than normal weight, it significantly reduced the dead load of the slab by 20%. Since the dead load was lighter, the number of post-tensioning cables and the amount of reinforcement for the typical frame was reduced. Unfortunately, deflection limited this slab to a minimum of 8”. As stated before, the maximum building height provided with a 7 ½” slab was only ½” from the allowable height, which means that our building would be greater than allowed. This is the case for all of the alternate systems when I completed my analytical research. This alternative design is not completely thrown out the window though. There was a great deal of reduction in the amount of cables and reinforcement and also the total weight of the building was reduced. These factors would result in smaller lateral resisting elements and foundations. A more in depth investigation dealing with the amount of rebar, post-tensioning cables, foundation and lateral element sizes may offset the cost of the lightweight concrete and the loss of one floor to be under the allowable height.

The next system that was analyzed was the Girder-Slab system. This system is not very feasible unless the architectural floor plan in the building is completely changed. When I completed the analysis, I found that the DB beams would not work for the largest span of the typical bay and also this system is only economical for typical bays. This system would only be an option if the bays were maximized at 20’ X 22’, the architectural floor plan completely changed to become typical, and the cheap cost of using this system would have to counter the loss of one floor to maintain allowable height. All of these factors make this option unrealistic.

Another system that had a potential for achieving a thin slab was a Two-Way Flat Plate. This system was analyzed with normal weight and lightweight concrete. The difference between the normal and lightweight concrete was not very significant. The overall cost of lightweight concrete compared to normal weight concrete would most likely not counter the amount of steel that was required by the normal weight system. Also the slab thickness was limited to 11” due to the deflection criteria. Though this system is relatively easy to build and requires minimum formwork, it requires the unreasonable changes of the floor plan because this system works with typical bays, which Wisconsin Place Residential does not contain. Also the loss of a floor would have to be taken into consideration in order to stay within the restricted height.

The final alternative system that was researched was a composite steel deck supported by non-composite steel beams. Before the analysis was conducted, this system seemed unreasonable. Steel construction is not very common in the Washington, D.C. area and also using steel results in larger floor-to-floor heights. After the analysis was completed, the floor girders resulted in a 17.5” depth. The depth, the requirement of additional fire proofing, the potential vibration issues, and the lead time will make this system economically unfeasible.

When comparing the four alternative floor systems and comparing them to the existing building, the analysis showed that the existing Two-Way Post-Tensioned Flat Plate system remains the most economic solution. All of the systems with the exception of the Two-Way Post-Tensioned Flat Plate w/ Lightweight Concrete require the floor plan to be significantly changed into a typical grid, which will limit the architecture of the building. Also, most of the lateral and foundation elements will have to be completely redesigned and the building will lose at least one floor. All of these changes are economically unfeasible and the building should remain unchanged with a Two-Way Post-Tensioned Flat Plate system.

Existing Structural System

Foundations

The foundation shall be supported on spread footings. Column and wall footings supported by rock shall be designed for a bearing pressure of 40,000 PSF. A 4-inch gravel base shall be provided below floor slabs as a moisture barrier. Also, under-floor sub-drainage system shall be installed. All exterior footings shall be a minimum of 2'-6" below grade. All controlled compacted fill shall be compacted to not less than 95% of the maximum dry density determined in accordance with ASTM D-698.

Floor Systems

<u>1^{st} Floor:</u>
Slab on grade.

<u>2^{nd} - 12^{th} Floor:</u>
Flat plate 7 ½" thick unbounded post-tension slabs, with a two-way bottom reinforcement mat of #4@24" continuous bars each way. Hooked bars at discontinuous ends are provided along with 2 #5 top and bottom additional bars along free slab edges. Concrete for slabs shall be normal weight concrete at 5000 psi. The post-tension cables consist of uniform tendons being pulled in the S-N direction and the banded tendons are in the pulled in the W-E direction of the building. The typical uniform cables are 15.0 klf and the banded cables range from approximately 50 - 400 kips.

<u>13^{th} Floor:</u>
Floors are typically post-tensioned the same as the 2^{nd} - 12^{th} except in the pool area. The 12" and 15" slab areas require #5@24" O.C. each way continuous on

top and bottom. The 23” slab area requires #6@12” O.C. each way continuous on top and bottom.

Pool House Roof:

7” slab with normal weight concrete and 60,000 psi reinforcing steel. A top and bottom mat of #4@12” O.C. continuous each way is required. Additional top reinforcing for column and middle strips is 6#5 top bars.

14th and 15th Floors:

Floors are typically post-tensioned the same as the 2nd - 12th.

Main Roof:

Slab is 8” thick unbounded post tensioned with a two-way bottom reinforcement of #4@24” continuous each way. For the 10” and 12” thick areas, #5@24” continuous mats are required as well as 2 #6 top and bottom additional bars along free slab edges.

Columns

The columns in Wisconsin Place Residential are primarily standard reinforced concrete with varying sizes, shape, and reinforcement depending on their location and loads that are applied throughout the building. The most typical shapes are 16”x28” and 16”x32”. The reinforcement for the columns varies from floor to floor. The typical reinforcement is 8#7 or 8#8 bars, but varies throughout typical levels. The 12th – 13th floor reinforcement is typically #10 or #11 bars, due to the fact that they are supporting the pool. The loads vary greatly from column to column and are as large as 1380k and as small as 122k for dead loads and 293k to 17k for live loads at the top of the pad.

Alternative Structural Systems

The structural layout of Wisconsin Place Residential is that of a very irregular building. There is not a typical bay due to the architectural layout of the condominiums as shown in **Figure: 3**. However I have chosen to design the alternative systems as if they were for a typical frame or bay for simplicity. **Figure: 1** shows the existing condition, where as **Figure: 2** shows the assumption I made to perform my analysis. By not having typical bays in the building, that significantly reduces the amount of alternative systems that would be able to work without a change in the architectural floor plans. As of right now, the building is 15 stories high with a building height permitted at 143’. The existing post-tensioned system of Wisconsin Place Residential is providing 142’-11 ½ “, which is only ½” less than the allowable height. This makes an alternative design very difficult since the floors are only 7 ½” thick.

This section will summarize the results of the alternative structural systems and compare the advantages and disadvantages of each under consideration.

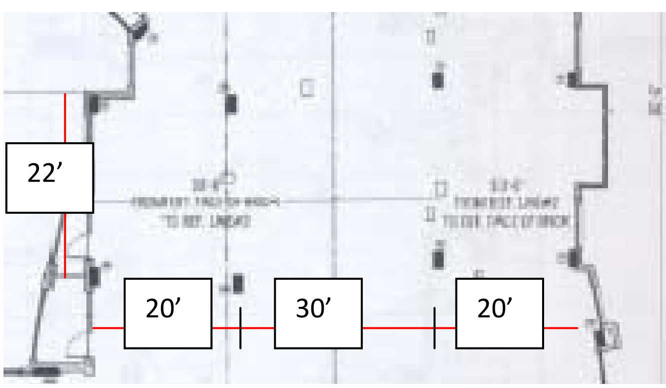


Figure: 1 (Actual Existing Frame)

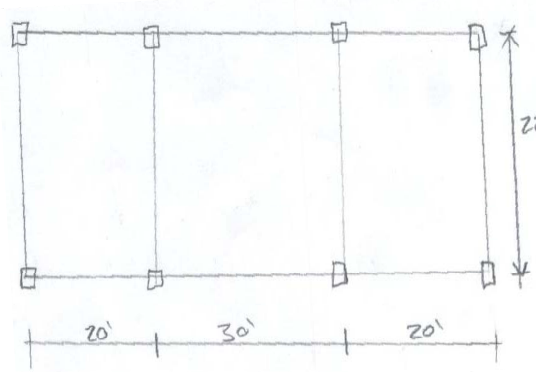


Figure: 2 (Frame Used for Analysis)

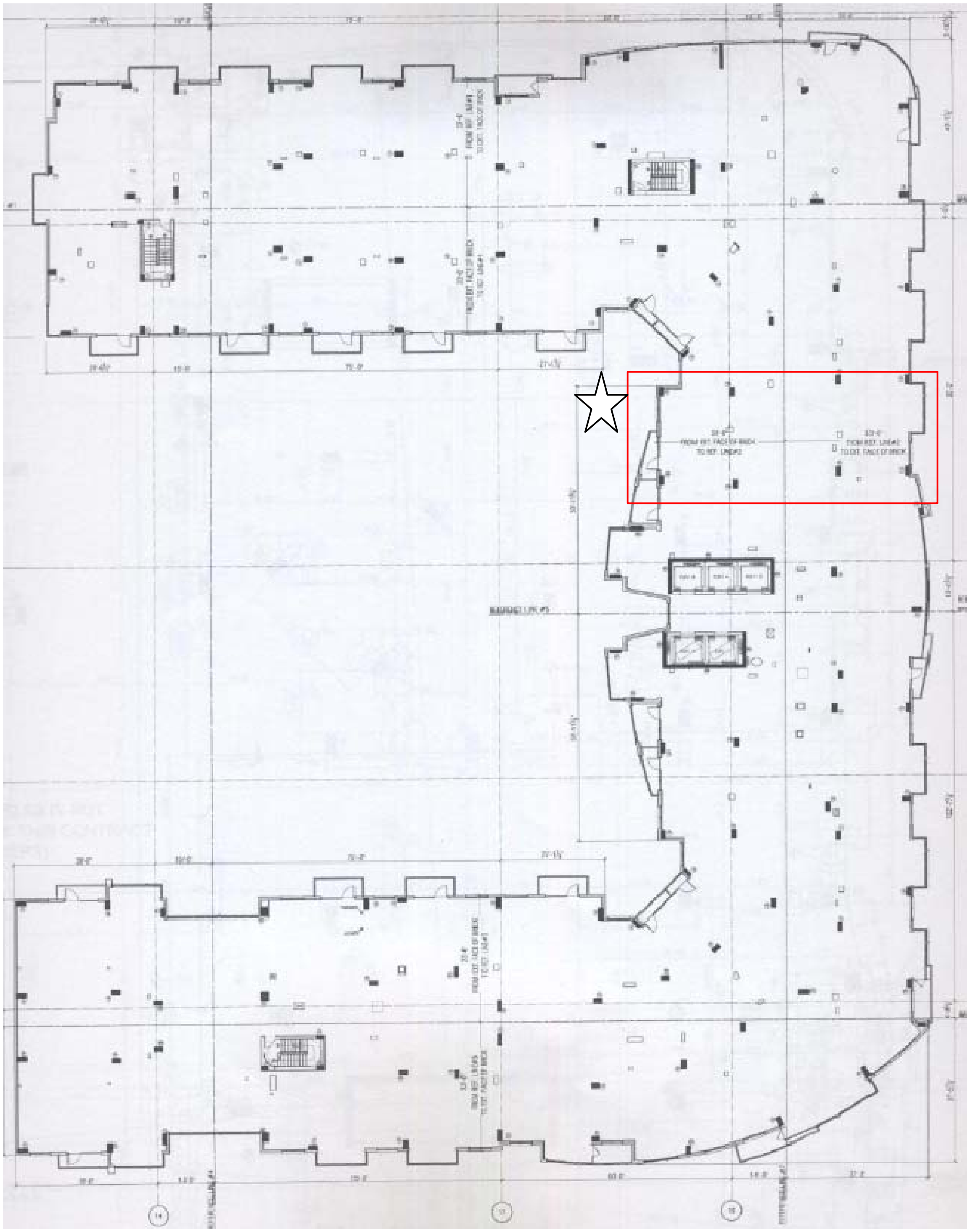


Figure: 3 (The rectangular area marked with a "☆ " is the typical frame used to design the alternative floor systems throughout this report and is blown up in **Figure: 1**

Redesign of Two-Way Post-Tensioned Flat Plate w/ Lightweight Concrete

A two-way post-tensioned flat plate system is one of the best ways of achieving the thinnest slab with larger spans. The existing slab depth was 7 1/2 “, however when redesigned with lightweight concrete the deflection criteria determined that an 8” slab was the minimum. The reason for this thicker slab using lightweight concrete is most likely because the design is done by hand, instead of a computer model. The floor height is a concern, because there is a maximum height permitted. From an economical stand point the goal is to make the building as light as possible with the most floors within the maximum height. Using lightweight concrete results in better thermal properties, better fire ratings, less micro-cracking as a result of better elastic compatibility, and better shock and sound absorption.

The alternative redesign resulted in a reduction of 405k (16 cables) to 266k (10 cables) for the typical frame. The rebar was also reduced over the column supports from 8#6 to 9#4 bars. Please note when analyzing the moments in this frame I assumed three equal 30’ bays instead of what is shown in **Figure: 4**. This was done to save time of creating a computer model and/or performing moment distribution. I am being extremely conservative, but using the AISC Steel Manuel will yield close enough results for the preliminary analysis this report requires. By using lightweight concrete, this will reduce the seismic lateral forces due to the weight of the building, reduce the number of cables needed due to the balancing of moments, and also improve the overall fire rating.

*See Appendix for supporting calculations

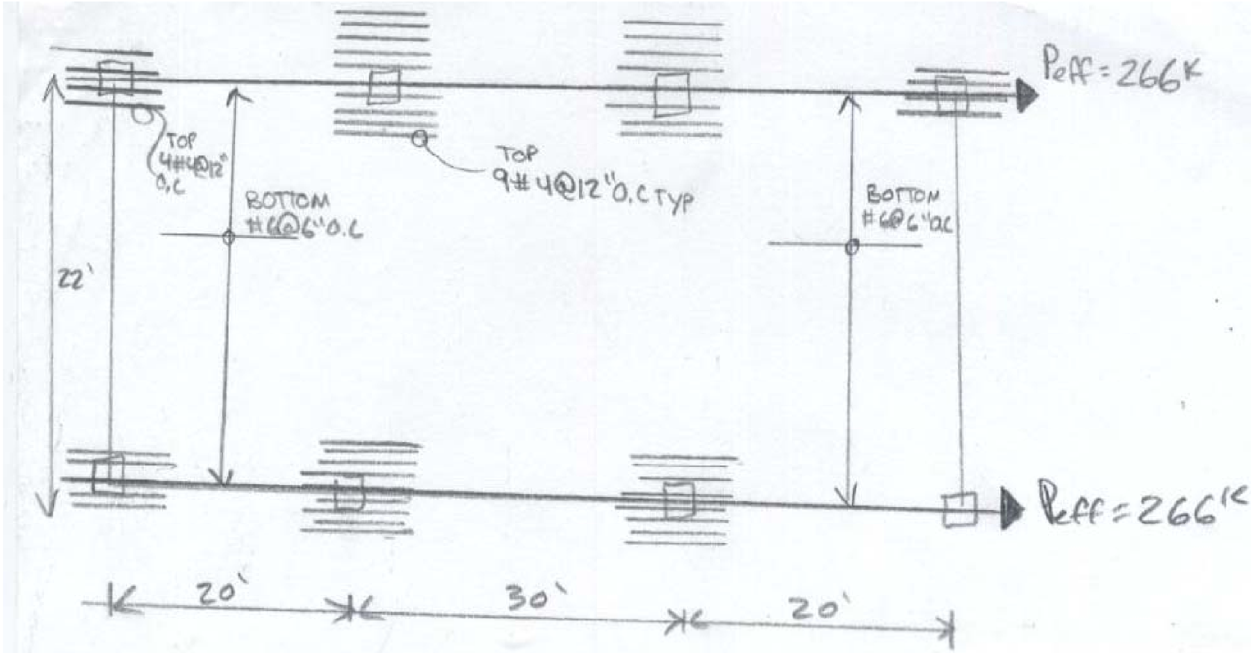


Figure: 4

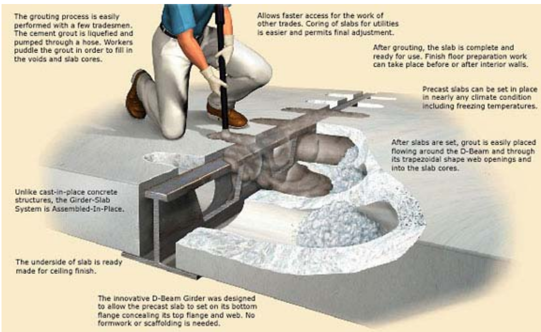
Advantages

Typically used in effort to create the thinnest slabs, achieve longer clear spans, fewer beams, and more slender elements. By having thinner slabs this will result to lower foundation cost and can be a great advantage in seismic regions. Lower building heights will result in savings for mechanical systems and façade costs. Also with post-tensioning the beams and slabs can be continuous.

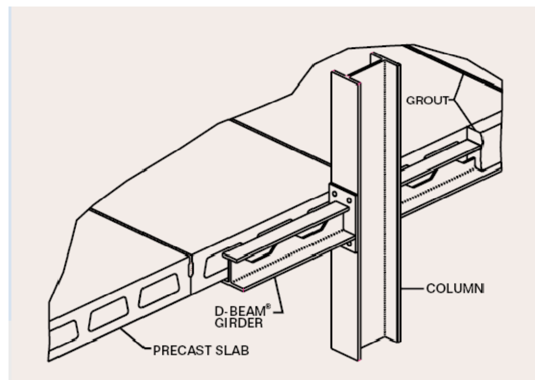
Disadvantages

For large spans, flat plate slabs with no drop panels are uneconomical due to the additional reinforcement required about columns. Post-tensioning systems can be difficult and time consuming if not a familiar practice commonly used by the contractor. Once the floors are placed the cables need to be stressed and the calculated elongations need to be checked by the engineer of record before they may be cut. All this adds more time to the process of building, hence more expensive.

Precast Girder-Slab



A Precast Girder-Slab system is a steel and precast hybrid system that forms a monolithic structural slab assembly. A special steel beam is used as an interior girder supporting the precast slab on its bottom flange. The flat structural slab permits minimum and variable floor-to-floor heights. When designing the hollow-core floor planks I referenced the tables created by Nitterhouse Concrete Products. The tables that I used are in the Appendix. The plank resulted in being 6”X 4’-0” with a 2” topping spanning a distance of 22’-0” with a 2hr fire resistance. The topping strength required was 8,000psi. Supporting these hollow-core planks are the special DB beams found on www.girder-slab.com. DB 8 X 42 girders spanning 20’ were required. Please note that I used the smaller spanning bay because DB beams would not work for the 30’ X 22’ bay. This means that in order for this system to work the floor plan would need to be altered. The hollow-core planks are capable of being used, but the depth of the floor will significantly increased without the use of DB beams. The typical bay used for this analysis can be seen in **Figure: 5** *See Appendix for supporting calculations



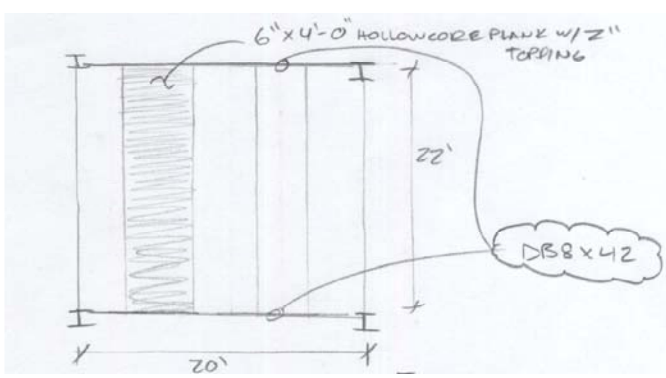


Figure: 5 (Typical Bay for Girder-Slab System)

Advantages

- Low floor-to floor heights
- Fast structure and building completion
- Reduced building weight, hence lower seismic forces
- Flexible floor plans
- Structure assembly is one process that limits on-site labor

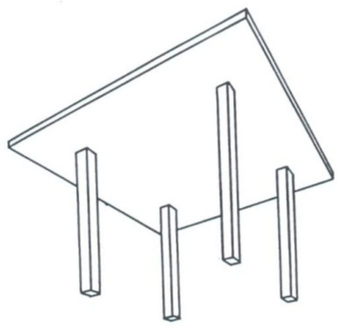
Disadvantages

- Limited available DB shapes
- Only 2-3 stories can be erected before grouting is required.
- Fire Proofing is required

Typical Girder-Slab Structure

Two-Way Flat Plate w/ Normal and Lightweight Concrete

picture 1



A Two-way flat plate system is a very common structural alternative in the Washington D.C. area. The minimum slab thickness required was governed by the deflection criteria and resulted in an 11" slab. This additional 3 ½ " of concrete would increase the weight of the building significantly, thus the seismic forces would be larger. Because the same strength can be achieved using lightweight or normal weight concrete, an analysis of both were performed. Using lightweight concrete has certain advantages as mentioned before, but on the downside it is more expensive than normal weight concrete. The analysis showed that by using normal weight concrete rather than lightweight concrete resulted in minimizing all reinforcement spacing by approximately 2". This would result in about 16 extra bars throughout a typical bay. The reinforcement required for the normal weight and lightweight concrete can be seen in the figures below. *See Appendix for supporting calculations

LONG SPAN	STRIP	Mu (Ft-K)	POS.	b (in)	d (in)	Mn (Ft-K)	R	ρ	As (in²)	Reinforcement: Size	N	Nmin	S	Smax
30'	COLUMN .75Mu⁻	312	NEG	132	10	347	316	.0055	7.26	#4	37	6	3	18
	.60Mu⁺	134	POS	132	10	149	136	.0025	3.3	#4	17	6	7	18
	MIDDLE .25Mu⁻	104	NEG	132	10	116	106	.0020	2.64	#4	14	6	9	18
	.40Mu⁺	89	POS	132	10	99	90	.0017	2.24	#4	12	6	11	18
SHORT SPAN 22'	COLUMN .75Mu⁻	201	NEG	180	9.5	224	166	.0050	5.13	#4	26	9	6	18
	.60Mu⁺	87	POS	180	9.5	97	72	.0015	2.57	#4	13	9	13	18
	MIDDLE .25Mu⁻	67	NEG	180	9.5	75	56	.0016	1.71	#4	9	9	20	18
	.40Mu⁺	57	POS	180	9.5	64	46	.0010	1.71	#4	9	9	20	18

Figure: 6 (Reinforcement Required for Normal Weight Concrete)

LONG SPAN	STRIP	POSITION	Mu (Ft-K)	b (in)	d (in)	Mn (Ft-K)	R	ρ	As (in²)	Reinforcement: SIZE	N	Nmin	S	Smax
30'	COLUMN .75Mu⁻	NEG	263	132	10	293	267	.0050	6.6	#4	33	6	4"	18
	.60Mu⁺	POS	113	132	10	126	115	.0020	2.64	#4	14	6	9"	18
	MIDDLE .25Mu⁻	NEG	88	132	10	98	90	.0018	2.38	#4	12	6	11"	18
	.40Mu⁺	POS	76	132	10	85	78	.0015	1.98	#4	10	6	13"	18
SHORT SPAN 22'	COLUMN .75Mu⁻	NEG	170	180	9.5	189	140	.0025	4.28	#4	22	9	8"	18
	.60Mu⁺	POS	73	180	9.5	82	61	.0013	2.22	#4	12	9	15"	18
	MIDDLE .25Mu⁻	NEG	57	180	9.5	64	48	.0010	1.71	#4	9	9	18"	18
	.40Mu⁺	POS	49	180	9.5	55	41	.0010	1.71	#4	9	9	18"	18

Figure: 7 (Reinforcement Required for Lightweight Concrete)

Advantages

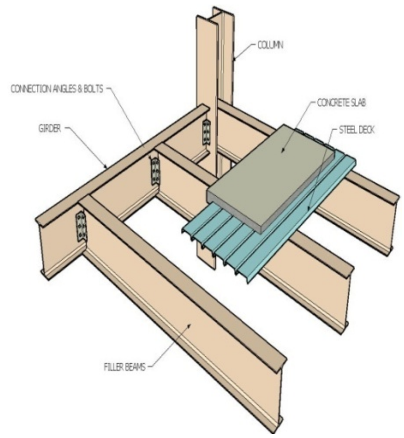
- Ease of constructability due to minimum formwork
- Two-Way slabs carry load in two directions, thus smaller supporting elements are required
- Exposed flat ceilings

Disadvantage

- Increasing the slab thickness causes the dead load weight to increase, thus bigger foundations and lateral members.
- Low Shear Capacity
- Low Stiffness

Composite Deck with Non-Composite Steel Framing

Composite Decking with Non-Composite Steel Framing is by far the least feasible alternative design to Wisconsin Place Residential. This system resulted with a 4 ½" Slab w/ 19 Gage, 2" LOK Floor w/ W1.4X1.4 WWF. The decking is composite decking from the USD Decking Manual. The beams supporting the deck are W12 X 26. The girders are extremely heavy and resulted in W12 X 170. This girder was not the most economical, but the total height of the building is the most critical part of the design. Therefore, I wanted to keep the depth to a minimum and a W12 X 170 had the least depth out of W-Shapes that would work. The deflection of the girder was the controlling factor in the design. This system is not very efficient for the long spans that are proposed in **Figure: 8** and would most likely require the spans to be reduced in half. Reducing the spans was not investigated, because the architecture would have to be completely redesigned and the overall depth achieved with this system with long spans is 17.5". This system also has potential issues with vibration and also requires fire-proofing, all which increase the cost of the building. Also If this system is used the lateral system would have to be completely designed because you would not be able to have shear walls. The total weight of the building would be reduced however, and the foundation would be significantly decreased.



Advantages

- Fast Construction
- Light Structural System
- Availability of Shapes

Disadvantages

- Potential Vibration
- Long lead time required
- Fire-Proofing is required
- Small Spans
- Large Floor-to-Floor heights

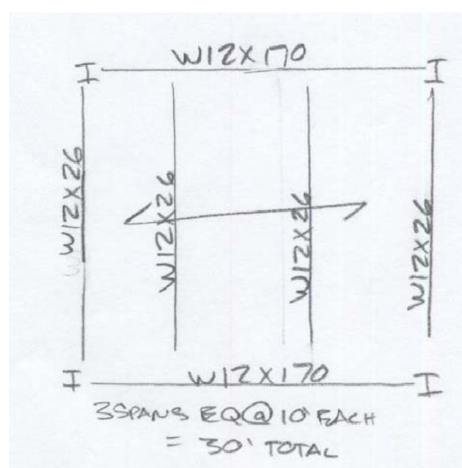


Figure: 8

Conclusions

After completing the analysis of the four alternate systems, each system had unique advantages and disadvantages. When comparing the four alternative floor systems and comparing them to the existing building, the analysis showed that the existing Two-Way Post-Tensioned Flat Plate system remains the most economic solution. All of the systems with the exception of the Two-Way Post-Tensioned Flat Plate w/ Lightweight Concrete require the floor plan to be significantly changed into a typical grid, which will limit the architecture of the building. Also, most of the lateral and foundation elements will have to be completely redesigned and the building will lose at least one floor. All of these changes are economically unfeasible and the building should remain unchanged with a Two-Way Post-Tensioned Flat Plate system.

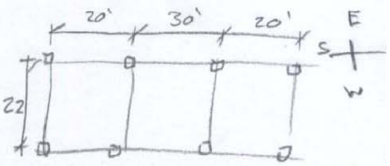
System	Two-Way Post-Tensioned Flat Plate w/ Normal Weight Concrete (EXISTING)	Two-Way Post-Tensioned Flat Plate w/ Lightweight Concrete	Precast Girder-Slab	Two-Way Flat Plate w/ Normal Weight Concrete	Two-Way Flat Plate w/ Lightweight Concrete	Composite Deck with Non-Composite Steel Framing
Weight (psf)	94	74	74	138	101	45
Slab Depth (in)	7.5	8	6	11	11	3.5
Largest Depth	7.5	8	8	11	11	17.5
Construction Difficulty	Hard	Hard	Easy	Easy	Easy	Easy
Lead Time	Short	Short	Long	Short	Short	Long
Formwork	Yes	Yes	Little	Little	Little	Little
Additional Fireproofing	No	No	Yes	No	No	Yes
Lateral System Effects	N/A	Medium	Medium	High	Medium	High
Relative Vibration	Low	Low	Medium	Low	Low	High
Foundation Impact	N/A	Medium	Medium	High	Medium	High
Cost/SF						
Materials	$10.62	$10.75	$10.72	$7.64	$7.77	$16.61
Labor	$8.01	$8.01	$3.15	$8.10	$8.10	$7.73
Total ($)	$18.63	$18.76	$13.87	$15.74	$15.87	$24.34
Viable Alternative	N/A	Maybe	No	No	No	No

Appendix: A

(Redesigned Two-Way Post-Tensioned Flat Plate w/ Lightweight Concrete)

TWO-WAY POST-TENSIONED REDESIGN W/ LWC. 1/

LOADS:



FRAMING DEAD LOAD = SELF WT

SUPERIMPOSED DEAD LOAD: PARTITIONS = 20 PSF

FINISHES & MISC = 5 PSF

MEP = 10 PSF

COLUMNS + WALLS = 10 PSF

TOTAL = 45 PSF

LIVE LOAD = 40 PSF (RESIDENTIAL)

2hr FIRE RATING

CONCRETE: LWC = 110 PCF

$f'_c = 5{,}000$ psi

$f'_{ci} = 3{,}000$ psi

Rebar: $f_y = 60{,}000$ psi

PT: UNBONDED TENDONS

1/2" Ø, 7-WIRE STRANDS $A = .153 \text{ in}^2$

$f_{pu} = 270$ ksi

ESTIMATE PRESTRESS LOSSES = 15 ksi (ACI 18.6)

$f_{se} = .7(270) - 15 = 174$ ksi (ACI 18.5.1)

$P_{eff} = A * f_{se} = (.153)(174) = 26.6$ k/tendon

DETERMINE PRELIM SLAB THK, $L/H = 45$

LONGEST SPAN = 30'

$h = \frac{(30')(12)}{45} = 8.0"$ prelim SLAB THK.

LOADING:

$DL = SW = \left(\frac{8 in}{12}\right)(110) = 73.3$ PSF (L.W.C.)

$SIDL = 45$ PSF

$LL_o = 40$ PSF

DUE TO BAYS NOT ACTUALLY BEING EQUAL, I DID NOT REDUCE THE LIVE LOAD IN ORDER TO BE ON THE CONSERVATIVE SIDE.

DESIGN OF S-N INTERIOR FRAME USING Equivalent FRAME method.

TOTAL BAY WIDTH BETWEEN CENTERLINES = 22'

IGNORE COLUMN STIFFNESS IN EQUATIONS FOR SIMPLICITY OF HAND CALCULATIONS.

cont'd on next page

* NO PATTERN LOADING REQ'd SINCE $LL/DL < 3/4$ 2/11

$40/73.3 = .54 < .75$ ✓OK

(ACI 13.7.6)

CALCULATE SECTION PROPERTIES

TWO-WAY SLAB MUST BE DESIGNED AS CLASS U (ACI 18.3.3) GROSS CROSS-SEC. PROPERTIES ALLOWED.

$A = bh = (22')(12)(8\text{ in}) = 2112\text{ in}^2$

$S = \frac{bh^2}{6} = \frac{(264)(8)^2}{6} = 2816\text{ in}^3$

SET DESIGN PARAMETERS

At time OF JACKING

$f'_{ci} = 3{,}000\text{ psi}$

compression $= .6 f'_{ci} = .6(3000) = 1800\text{ psi}$

Tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164\text{ psi}$

Avg. precompression limits

$P/A = 125$ psi min

300 psi max

TARGET LOAD BALANCES

USE $.75 w_{DL} = .75(73.3) = 55$ PSF

COVER REQUIREMENTS (2-hr) ASSUME CARBONATE AGGREGATE

RESTRAINED SLABS = 3/4" bot

UN RESTRAINED " 1 1/2" bot

BOTH = 3/4" TOP

TENDON ORDINATE	TENDON (CG) LOCATION
EXT. SUPPORT - anchor	4.0"
INT. SUPPORT - top	7.0"
INT. SPAN - bot	1.0"
END SPAN - bot	1.75"

Cont'd on next page.

3/11

$a_{INT} = 7.0" - 1.0" = 6.0"$

$a_{end} = (4.0 + 7.0)/2 - 1.75 = 3.75$

ℓ varies along the span.

PRESTRESS FORCE REQ'd TO BALANCE 75% OF S.W. DL.

$w_b = .75 w_{DL} = .75(73.3)(22') = 1.209\ k/ft$

FORCE NEEDED IN TENDONS TO COUNTERACT THE LOAD IN BAY

EXT

$$P = \frac{w_b L^2}{8 a_{end}} = \frac{(1.209)(20')^2}{8(3.75/12)} = 193.44^k$$

INT.

$$P = \frac{(1.209)(30')^2}{8(6/12)} = 272.025^k$$

INTERIOR SPAN CONTROLS

CHECK PRECOMPRESSION ALLOWANCE

DETERMINE NUMBER OF TENDONS TO ACHIEVE 273^K

$$\#\ TENDONS = \frac{273}{26.6} = 10.26$$

USE 10 TENDONS

ACTUAL FORCE FOR BANDED TENDONS

$P_{ACT} = (10\ tendons)(26.6^k) = 266^K$

THE BALANCED LOAD FOR SPAN IS SLIGHTLY ADJUSTED

$w_b = (266/273)(1.209) = 1.18\ k/ft$

DETERMINE ACTUAL PRECOMPRESSION STRESS

$$\frac{P_{ACT}}{A} = \frac{266^k(1000)}{2112} = 125.947\ psi > 125\ psi\ min\ ✓OK$$

$< 300\ psi$ MAX ✓OK

CHECK EXT. SPAN

$P = 193.44^k < 266^k$ LESS FORCE REQ'd IN EXT BAY.

Cont'd on next page

ASSUME CONTINUATION OF THE FORCE REQ'd FOR END SPANS INTO INTERIOR SPANS & CHECK AMOUNT OF LOAD THAT WILL BE BALANCED 9/11

$w_b = (266)(8)\left(\frac{3.75}{12}\right) / 20^2$

$w_b = 1.66 \, k/ft$

$w_{DL} = (.074 \, KSF)(22') = 1.63 \, k/ft$

$w_b / w_{DL} = \frac{1.66}{1.63} = 102\%$; FOR HAND CALCULATIONS THIS WILL BE ACCEPTABLE FOR THIS DESIGN. BECAUSE IF YOU DROP TO 9 CABLES YOU WONT MEET THE MIN 125 PSI REQ'D & 10 CABLES WILL MAKE THE % HIGHER. YOU CAN CHANGE THE CABLE PROFILE POINTS FOR A MORE ACCURATE LOADING.

S-N
INTERIOR
FRAME

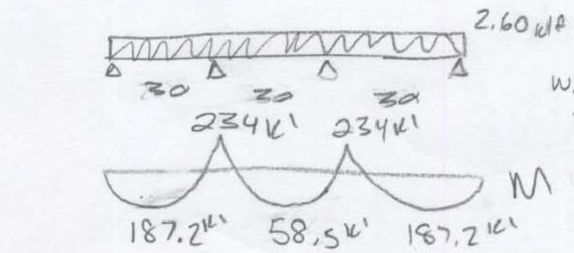
EFFECTIVE PRESTRESS FORCE, $P_{EFF} = 266^k$

CHECK SLAB STRESSES

* DEAD LOAD MOMENTS *

$w_{DL} = 73.3 \, PSF \, (SW) + 45 \, (SIDL) = 118.3 \, PSF$

$\frac{(118.3)(22')}{1000} = 2.60 \, k/f$



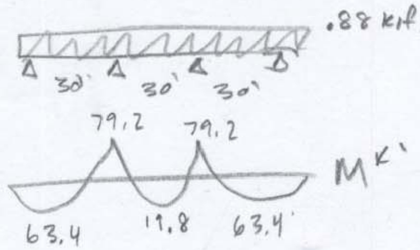
$wl^2 = (2.60)(30)^2$
$2340 \, k'$

NOTE: TO REDUCE TIME OF DOING MOMENT DISTRIBUTION I USED TABLE 3-22c OF STEEL MANUAL
MOMENT & SHEAR COEFF. EQUAL SPANS, EQUAL LOADED BEAMS.
I ASSUME ALL BAYS TO BE 30' WHICH IS EXTREMELY CONSERVATIVE.

LIVE LOAD MOMENTS

$W_{LL} = \frac{40PSF(22')}{1000} = .88 \text{ klf}$

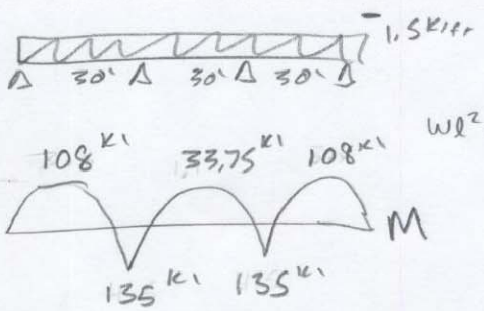
$wl^2 = .88(30)^2 = 792^{k'}$



TOTAL BALANCING MOMENTS, M_{bal}

w_b = AVERAGE OF 3 BAYS.

$\frac{1.18 + 2(1.66)}{3} = -1.5 \text{ klft}$



$wl^2 = (-1.5)(30)^2 = 1350$

STAGE I: STRESSES IMMEDIATELY AFTER JACKING (DL + PT) ACI 18.4.1

MIDSPAN STRESSES

$f_{TOP} = \frac{(-M_{DL} + M_{bal})}{S} - P/A$

$f_{bot} = \frac{(+M_{DL} - M_{bal})}{S} - P/A$

INT. SPAN

$f_{TOP} = \frac{(-58.5 + 33.75)(12)(1000)}{2816} - 125.947$

$f_{TOP} = -231.416 < .60(f'_{ci}) = 1800 \text{ psi}$ ✓OK.

$f_{bot} = \frac{(58.5 - 33.75)(12)(1000)}{2816} - 125.947$

$f_{bot} = -22.61 < 1800 \text{ psi}$ ✓OK.

END SPAN

6/11

$$f_{TOP} = \frac{(-187.2 + 108)(12{,}000)}{2816} - 125.947 = -463.4 < 1800\text{psi} \checkmark \text{OK}$$

$$f_{bot} = \frac{(187.2 - 108)(12{,}000)}{2816} - 125.947 = 211.55 > 164$$, but please note moments are alot higher due to increased spans of 3=30' instead of actual 20' - 30' - 20'. Three EQ spans at 30' would require the $f'_{ci} = 5{,}000$ psi! Full strength before being jacked. Assume this is acceptable for this example.

SUPPORT STRESSES

$$f_{top} = \frac{(+M_{DL} - M_{bal})}{S} - P/A$$

$$f_{bot} = \frac{(-M_{DL} + M_{bal})}{S} - P/A$$

$$f_{top} = \frac{(234 - 135)(12{,}000)}{2816} - 125.947$$

$f_{TOP} = 295.9 > 3\sqrt{f'_{ci}}$, but refer to (note above)

$$f_{bot} = \frac{(-234 + 135)(12000)}{2816} - 125.947$$

$f_{bot} = -547.822 < 1800 \checkmark$ OK

STAGE 2.

STRESSES AT SERVICE LOAD (DL + LL + PT)

MIDSPAN STRESSES

$$f_{TOP} = \frac{(-M_{DL} - M_{LL} + M_{bal})}{S} - P/A$$

$$f_{bot} = \frac{(+M_{DL} + M_{LL} - M_{bal})}{S} - P/A$$

INT SPAN

$$f_{TOP} \quad \frac{(-58.5 - 19.8 + 33.75)(12000)}{2816}$$

$$-189.84 - 125.947 = -315.8$$

$f_{TOP} = -315.8 < .45f'_c = 2250 \checkmark$ OK.

7/1

$$f_{bot}\ \frac{(58.5 + 19.8 - 33.75)(12{,}000)}{2816} - 125.947$$

$f_{bot} = 63.9 < 6\sqrt{f'_c} = 424\ psi$ ✓OK.

END SPAN

$$f_{top} = \frac{(-187.2 - 63.4 + 108)(12{,}000)}{2816} - 125.947$$

$f_{Top} = 733.617 < .45\ f'_c = 2250\ psi$ ✓OK

$$f_{bot} = \frac{(187.2 + 63.4 - 108)(12000)}{2816} - 125.947$$

$f_{bot} = 481.723 > 6\sqrt{f'_c} = 424$; PLEASE REFER TO NOTE ON PAGE 6 OF CALCULATIONS FOR PT, ASSUME OK.

SUPPORT STRESSES

$$f_{TOP} = \frac{(+234 + 79.2 - 135)(12000)}{2816} - 125.947$$

$f_{Top} = 633.429 > 424\ psi$;

$$f_{bot} = \frac{(-234 - 79.2 + 135)(12000)}{2816} - 125.947 = -885.3 < .45 f'_c = 2250\ psi\ ✓OK$$

ASSUMING IF MOMENT DISTRIBUTION WAS USED MOMENTS WOULD BE LOWER AND ALSO SPANS ARE SHORTER, THEREFORE STRESSES SHOULD BE ACCEPTABLE, BUT FURTHER ANALYSIS NEEDS TO BE COMPUTED OR A COMPUTER PROGRAM NEEDS TO BE USED.

CONT'D ON NEXT PAGE.

8/11

Ultimate STRENGTH.

DETERMINE FACTORED MOMENTS.

$M_1 = P * e$

e: 0 at ext support

$e = 3.0$ at int support (UA → center of tendon)

$M_1 = \frac{(266)(3.0)}{12} = 66.5$

SECONDARY P-T MOMENTS M_{sec} VARY linearly b/w supports

$M_{sec} = M_{bal} - M_1$

$M_{sec} = 135 - 66.5$

$M_{sec} = 68.5$ at int supports.

$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec}$

At midspan: $M_u = 1.2(\overset{224.64}{187.2}) + 1.6(\overset{101.44}{63.4}) + 1.0(\overset{34.25}{34.25}) = 360.33$

At support: $M_u = 1.2(\overset{280.8}{234}) + 1.6(\overset{126.72}{79.2}) + 1.0(68.5) = -476.02$

DETERMINE MIN. BONDED REINF.

POSITIVE MOMENT REGION:

INT SPAN: $f_t = 63.9 < 2\sqrt{f'_c} = 2\sqrt{5{,}000} = 141$ ✓OK

NO POSITIVE REINF REQ'd (ACI 18.9.3.1)

EXT. SPAN. $f_t = 481.723 > 2\sqrt{f'_c} = 2\sqrt{5000} = 141$

MIN positive REINFORCEMENT REQ'd.

$y = (f_t/(f_t + f_c))h$

$= \left(\frac{481}{(481 + 733.617)}\right)8 = 3.17$

cont'd on next page

$N_c = \frac{M_{DL+LL}}{S} (.5) y (l_2)$

$N_c = \left[\frac{(187.2 + 63.4)(12)}{2816}\right] (.5)(3.17)(22')(12)$

$N_c = 446.85$

$A_{s,min} = N_c / .5 f_y$

$= 446.85 / .5(60ksi)$

$A_{s,min} = 14.9 in^2$

DISTRIBUTE THE POSITIVE MOMENT REINF. UNIFORMLY ACROSS THE SLAB-BEAM WIDTH AND AS CLOSE AS PRACTABLE TO EXTREME FIBER.

$A_{s,min} = (14.9 in^2)/(22')$

$= .677 in^2/ft \Rightarrow$ #8@12" = .79

OR

#6@6" EASY TO PLACE

$\frac{.79}{12''} = \frac{.44}{x}$

$x = 6.68$

$x = 6''$

USE #6@6 O.C BOTTOM, MIN LENGTH SHALL BE 1/3 CLR SPAN AND CENTRED IN POSITIVE MOMENT REGION.

NEG. MOMENT REGION

$A_{s\,min} = .00075 A_{cf}$ (ACI 18.9.3.3)

INT. Supports

$A_{cf} = max\ 8\left[\left(\frac{30+20}{2}\right)\right] \times 12 = 2400)$

$A_{s\,min} = .00075(2400) = 1.80 in^2$

$= 9-$#4 TOP $= 1.80 in^2$

EXT SUPPORTS

$A_{cf} = max,\ 8(20/2)(12)$

$A_{s\,min} = .00075(960) = .72 in^2$

$= 4$#4 TOP $= .80$

cont'd on next page

10/11

MUST SPAN a MIN of 1/6 CLR ON EACH SIDE OF SUPPORT
AT LEAST 4 - BARS REQ'd IN EACH DIRECTION.
PLACE TOP BARS W/ 1.5h away FROM FACE OF SUPPORT

$= 1.5(8) = 12''$

MAX bar spacing = 12'' (ACI 18.9.3.3)

CHECK MIN REINFORCEMENT

$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$

$A_{ps} = .153 in^2 \times$ tendons

$= .153(10) = 1.53 in^2$

$f_{ps} = f_{se} + 10{,}000 + f'_c bd / 300A_{ps}$ for slabs with $L/h > 35$

$174{,}000 + 10{,}000 + [5{,}000)(22*12)d] / 300 \; 1.53$

$184{,}000 + 2875.82d$

$a = \dfrac{(A_s f_y + A_{ps} f_{ps})}{.85 f'_c b)}$

At supports

$d = 8'' - 3/4'' - 1/4'' = 7''$

$f_{ps} = 184{,}000 + 2875.82(7) = 204{,}131 psi$

$a = \dfrac{[(1.80)(60ksi) + 1.53(204ksi)]}{.85(5)(22\times12)} = \dfrac{420.12}{1122} = .37$

$\phi M_n = .9\left[(1.80)(60) + 1.53(204))(7 - \frac{.37}{2})/12\right]$

$.9(420.12)(6.815)/12$

$= 214.734 < 476.02 \Rightarrow$ REINFORCEMENT FOR ULTIMATE STRENGTH GOVERNS.

> **9 - #4 TOP INTERIOR SUPPORTS EACH WAY**
> **4 - #4 TOP EXTERIOR SUPPORTS EACH WAY.**

AT MIDSPAN (ENDSPAN) 11/11

$d = 8 - 1\frac{1}{2} - \frac{1}{4}" = 6\frac{1}{4}"$

$f_{ps} = 184{,}000 + 2875.82(6.25) = 201{,}974$

$a = [(14.9)(60) + 1.53(202)] \,/\, .85(5)(22 \times 12) =$

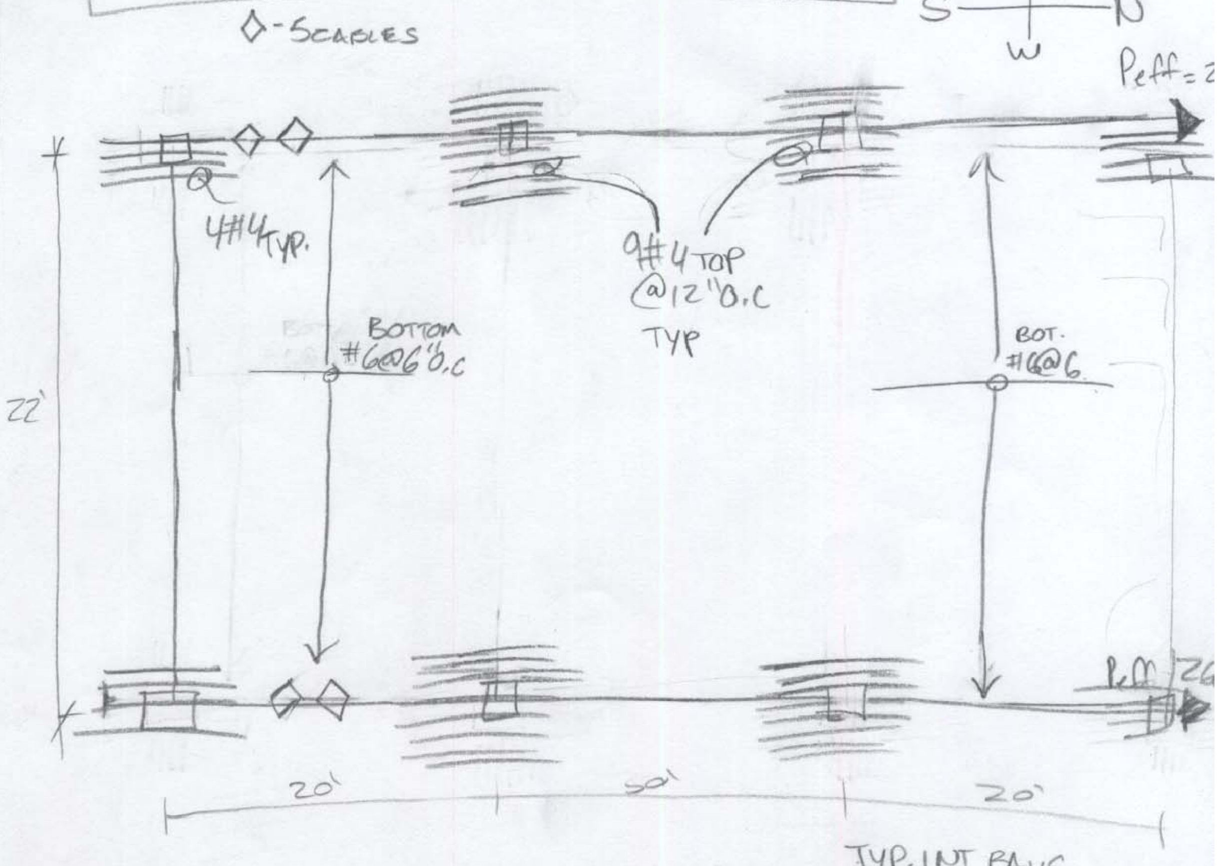
$1203.06 / 1122 = 1.07$

$\phi M_n = .9\,[14.9(60) + 1.53(202)]\,[6.25 - \frac{1.07}{2}]\,/12$

$.9(1203.06)(5.715)/12$

$\phi M_n = 515.67 > 360.33$ Min reinf. ✓OK

#6@6" O.C BOTTOM AT END SPANS.



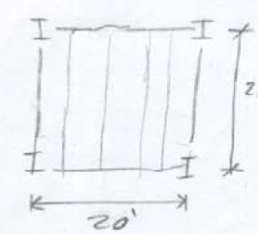
NOTE! YOU WILL NEED HOOK BARS AT EDGE OF SLABS. #5@12.

Appendix: B

(Precast Girder-Slab)

GIRDER-SLAB SYSTEM

1/5



DEADLOAD

PARTITIONS = 20PSF

FINISHES & MISC = 5PSF

MEP = 10PSF

35PSF

LIVELOAD

40PSF

*ASSUME NO LIVE LOAD REDUCTION BECAUSE I WANTED TO BE CONSERVATIVE.

PRECAST PLANK

$TL = (45 + 40) = 85PSF$ (DL, LL)

DL WT OF PLANK = 48.75PSF

TOPPING = 25PSF

PLANK = $f'c$ = 5ksi

ASSUME GROUT $f'c$=4ksi

SPAN 22'

USING NITTERHOUSE CONCRETE PRODUCTS PRESTRESSED CONCRETE

6" x 4'-0" HOLLOWCORE PLANKS

2HR FIRE RESIST. W/ 2" TOPPING

ALLOWABLE LOAD FOR 22' SPAN

4-1/2Ø => 86 PSF > 85PSF ✓OK

REFER TO ATTACHED TABLE FOR DB8X40 PROPERTIES & ALLOWABLE MOMENT

INITIAL LOAD

$$M_{DL} = \frac{(22')(.04875)(20')^2}{8} = 53.625^{K'}$$

(.04875 = WT OF PLANK)

TRY DB 8X40 ALLOW. MOMENT = 66K' > 53.625K' ✓OK

$$\Delta_{DL} = \frac{(5)(22')(.04875)(20)^4(1728)}{384(122)(29{,}000)} = 1.09''$$

TOTAL LOAD - COMPOSITE

• THE TRANSFORMED SECTION CARRIES THE SUPERIMPOSED LOADS & IS USED TO CALCULATE DEFLECTION

$$M_{SUP} = \frac{(22')(.035 + .040 + .025)(20')^2}{8} = 110^{K'}$$

(S.I.DL, LL, TOPPING)

$M_{TL} = 53.625 + 110 = 163.625$

$$S_{req} = \frac{(163.625)(12 in/ft)}{(.6)(50)} = 65.45 \leq 67.2 \text{ ✓OK}$$

cont'd on next page.

$\Delta_{SUP} = \frac{(5)(22)(.035+.04+.03)(20)^4(1728)}{384(289)(29{,}000)} = .94\,in$ $\quad L/240 = \frac{20 \times 12}{240} = 1\,in$ 2

$.94 < 1\,inch$ ✓ OK

CHECK COMPRESSIVE STRESS on CONCRETE

GROUT AT 4KSI

$N_{VALUE} = \frac{E_{STEEL}}{E_{CONCRETE}} = \frac{29{,}000\,ksi}{57{,}000\,(4{,}000)^{1/2}} = 8.04 \therefore S_{tc} = 8.04(67.2) = 540.3\,in^3$

$f_c = \frac{(110\,k')(12\,in/ft)}{540.3} = 2.44\,ksi \quad F_c = (.45)(4\,ksi) = 1.8\,ksi$

$1.8\,ksi < 2.44\,ksi \therefore$ NG

TRY GROUT @ 5 Ksi

$N_{value} = \frac{29{,}000}{57{,}000\,(5{,}000)^{1/2}} = 7.20 \therefore S_{tc} = 7.20(67.2) = 483.512$

$f_c = \frac{(110)(12)}{483.512} = 2.73 \quad F_c = .45(5) = 2.25 < 2.73 \therefore NG$

TRY 6KSi

$N = \frac{29{,}000}{57{,}000\,(6{,}000)^{1/2}} = 6.57 \therefore S_{tc} = 6.57(67.2) = 441.384$

$f_c = \frac{(110)(12)}{441.384} = 2.99 \quad F_c = .45(6) = 2.7$ N.G

TRY 7KSI

$N = \frac{29{,}000}{57{,}000\,(7{,}000)^{1/2}} = 6.08 \therefore S_{tc} = 6.08(67.2) = 408.576$

$f_c = \frac{(110)(12)}{408.576} = 3.23 \quad F_c = .45(7) = 3.15 < 3.23 \therefore NG$

cont'd on next page.

3/5

TRY 8KSI

$$N = \frac{29,000}{57,000\,(8,000)^{1/2}} = 5.69 \therefore \quad S_{tc} = 5.69(67.2) = 382.25$$

$$f_c = \frac{(110)(12)}{382.25} = 3.45 \qquad F_c = .45(8) = 3.6 > 3.45 \checkmark OK$$

CHECK BOTTOM FLANGE TENSIONSTRESS (TOTAL LOAD)

$$f_b = \frac{(53.625\,k\text{-}i)(12\,in/ft)}{(36.1\,in^3)} + \frac{(110\,k\text{-}i)(12)}{(67.9\,in^3)} = 37.27\,KSI$$

$$17.83 + 19.44$$

$$F_b = (.9)(50\,ksi) = 45\,ksi > 37.27\,ksi \checkmark OK.$$

CHECK SHEAR

$$\text{TOTAL LOAD} = (\overset{\text{PLANK}}{48.75} + \overset{\text{SIDL}}{35} + \overset{\text{L.L}}{40} + \overset{\text{TOP}}{25}) = 148.75\,psf$$

$$w = (.14875\,ksf)(22') = 3.27\,k/ft$$

$$R = \frac{(3.27\,k/ft)(20')}{2} = 32.7\,k$$

$$f_v = \frac{32.7\,k}{(.340)(3.5)} = 27.48\,ksi$$

$$F_v = (.4)(50\,ksi) = 20\,ksi < 27.48 \therefore N.G.$$

TRY USING

DB 8X42

cont'd on next page

4

TRY DB 8X42 ALLOW MOMENT = 66K' ≥ 53.625K' ✓OK

$$\Delta_{DL} = \frac{(5)(22)(.04875)(20^4)(1728)}{384(123)(29,000)} = 1.08''$$

DB8X42 * PROPERTIES REFER TO ATTACHE TABLES

TOTAL LOAD–COMPOSITE

M_{SUP} = 110K' SAME AS PREVIOUS CALCS.

M_{TL} = 163.625K' " " " "

$S_{req} = 65.45 \leq 67.5$ ✓OK

$$L/240 = \frac{20' \times 12}{240} = 1 \text{ inch}$$

$$\Delta_{SUP} = \frac{(5)(22)(.035+.04+.025)(20)^4(1728)}{384(291)(29,000)} = .939\text{in} < 1\text{inch} \checkmark OK$$

CHECK COMPRESSIVE STRESS ON CONCRETE

FROM PRIOR CALCS TRY GROUT AT 7KSI

$$N = \frac{29,000}{57,000(7,000^{1/2})} = 6.08 \quad S_{tc} = 6.08(67.5) = 410.4$$

$$f_c \; \frac{(110)(12)}{410.4} = 3.21 \qquad F_c = .45(7) = 3.15 < 3.21 \therefore N.G.$$

USE 8KSI MUST WORK BECAUSE WE HAVE BIGGER S_t, hence SMALLER f_c THAN BEFORE W/ THE D8X40.

CHECK BOT. FLANGE TENSION STRESS

$$f_b = \frac{(53.625K')(12 in/ft)}{36.9} + \frac{110K'(12)}{68.4} = 36.74 KSI$$

17.439 + 19.298

$F_b = .9(50) = 45Ksi \geq 36.74KSI$ ✓OK

CHECK SHEAR

TOT. LOAD = 148.75psf FROM PREV. CALCS

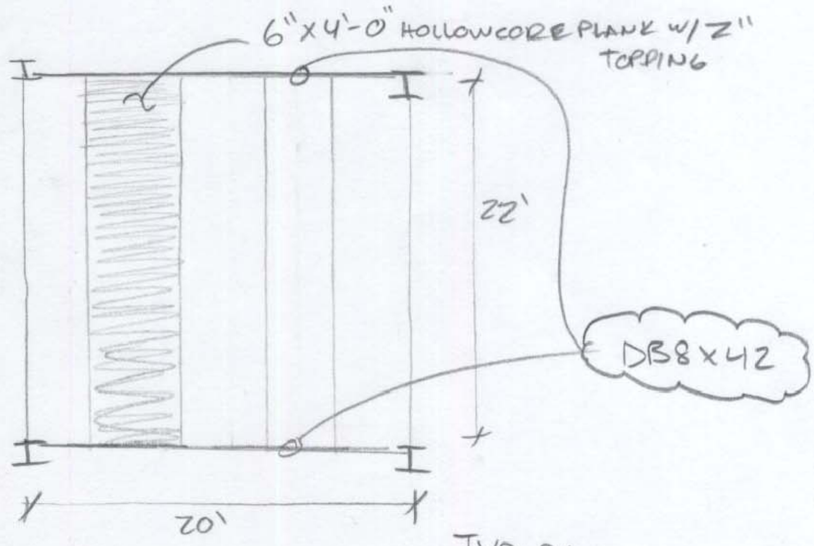
$w = 3.27$

$$R = \frac{(3.27^{k/ft})(20')}{2} = 32.7^{k}$$

$$f_v = \frac{32.7}{(.345)(5.5)} = 17.23$$

$F_v = .4(50ksi) = 20ksi > 17.23ksi$ ✓OK

USE 6" x 4'-0" HOLLOW CORE PLANKS SPANNING 22' W/ 2HR FIRE RESIST. W/ A 2" TOPPING HAVING A STRENGTH OF 8000psi. USE DB 8 x 42's SPANNING 20' TO SUPPORT THE PLANKS.



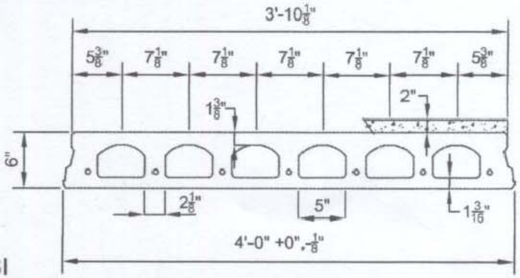
Prestressed Concrete
6"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 253 in.^{2}	Precast S_{bc} = 370 in.^{3}
I_c = 1519 in.^{4}	Topping S_{tc} = 551 in.^{3}
Y_{bc} = 4.10 in.	Precast S_{tc} = 799 in.^{3}
Y_{tc} = 1.90 in.	Wt.= 195 PLF
	Wt.= 48.75 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI.
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 67.5 k-ft
   7-1/2"Ø, 270K = 104.2 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c}$ = 580 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS																IBC 2003 & ACI 318-02 (1.2 D + 1.6 L)				
Strand Pattern		SPAN (FEET)																		
		11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
4 - 1/2"ø	LOAD (PSF)	227	187	360	306	268	229	194	165	141	120	102	86	73	61	50				
7 - 1/2"ø	LOAD (PSF)	367	305	495	455	418	387	340	312	275	243	215	189	167	147	130	114	97	83	70

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

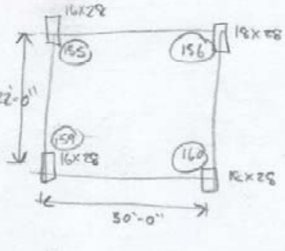
05/14/07

6F2.0T

Appendix: C

(Two-Way Flat Plate w/ Normal and Lightweight Concrete)

TWO-WAY FLAT PLATE W/ NORMAL WT CONCRETE



LOADS

DEAD

PARTITIONS 20PSF
FINISHES & MISC 5PSF
MEP 10PSF
COLUMNS & WALLS 10PSF
11" NW. SLAB - $150(11/12) = 138$PSF

LIVE
40PSF

TOTAL DEAD = 183 PSF
TOTAL LIVE = 40PSF

$f'_c = 5{,}000$ psi
$f_y = 60$ ksi

FLAT PLATE W/O DROP PANELS, INT PANELS (TABLE 9.5c) ACI 318-05

$$\frac{l_n}{33} = \frac{30' - \left(\frac{16+18}{2}/12\right)}{33} = .866' \times 12" = 10.39 \Rightarrow 11" \text{ SLAB}$$

FOR SHORT DIRECTION

$$W = 1.2D + 1.6L = 1.2(183) + 1.6(40) = 284\text{PSF}$$

$$M_o = \frac{wL^2}{8} = \frac{.284(30)(22' - 28/12)^2}{8} = 412^{FtK}$$

$$b_L = \frac{22'}{2} \times 12 = 132$$

$$b_S = \frac{30}{2} \times 12 = 180$$

$M_u^- = .65(412) = 268$ FtK

$M_u^+ = .35(412) = 144$ FtK

PLACE LONG-DIRECTION CLOSER TO TOP & BOT

$d_L = 11 - .75 - .5/2 = 10"$

$d_S = 10" - .5 = 9.5"$

FOR LONG DIRECTION

$$M_o = \frac{.284(22)(30 - 17/12)^2}{8} = 639^{FtK}$$

$M_u^- = .65(639) = 416^{FtK}$

$M_u^+ = .35(639) = 223^{FtK}$

$M_n = \frac{M_u}{\phi}$, $\phi = .9$

$N = \frac{A_s}{A_{s\,bar}}$

$R = \frac{M_n \times 12{,}000}{bd^2}$

$N_{min} = \frac{\text{WIDTH OF STRIP}}{2t}$

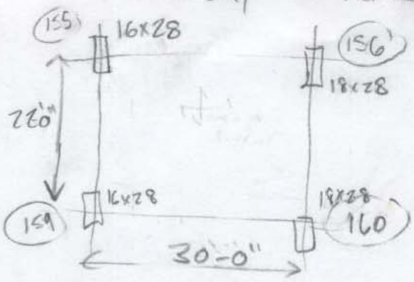
MAX. SPACING = 18"

$S = b/N$

$A_{s\,min} = .0018(11)(12) = .238\ in^2/ft$

				in	in	FtK			(in^2)	REINFORCEMENT				
LONG SPAN	STRIP	M_u (FtK)	POS.	b	d	M_n	R	ρ	As	SIZE	N	N_{min}	S	S_{MAX}
30'	COLUMN .75M_u^-	312	NEG	132	10	347	316	.0056	7.26	#4	37	6	3	18
	.60M_u^+	134	POS	132	10	149	136	.0025	3.3	#4	17	6	7	18
	MIDDLE .25M_u^-	104	NEG	132	10	116	106	.0020	2.64	#4	14	6	9	18
	.40M_u^+	89	POS	132	10	99	90	.0017	2.24	#4	12	6	11	18
SHORT SPAN 22'	COLUMN .75M_u^-	201	NEG	180	9.5	224	166	.0050	5.13	#4	26	9	6	18
	.60M_u^+	87	POS	180	9.5	97	72	.0015	2.57	#4	13	9	13	18
	MIDDLE .25M_u^-	67	NEG	180	9.5	75	56	.0010	1.71	#4	9	9	20	18
	.40M_u^+	57	POS	180	9.5	64	46	.0010	1.71	#4	9	9	20	18

TWO-WAY FLAT PLATE W/LIGHTWEIGHT CONCRETE



$f'_c = 5{,}000\,psi$

$f_y = 60\,ksi$

LOADS

DEAD

PARTITIONS = 20PSF

FINISHES & MISC = 5PSF

MEP = 10PSF

COLUMNS + WALLS = 10PSF

11" LIGHTWEIGHT SLAB = $110(11''/12) = 101\,PSF$
110PCF

TOTAL DEAD = 146PSF

TOTAL LIVE = 40PSF

LIVE

40PSF

THICKNESS OF SLAB

DEFLECTION LIMIT

FLAT PLATE W/O DROP PANELS, INT PANELS (TABLE 9.5c) ACI 318-05

$$\frac{l_n}{33} = \frac{30' - \left(\frac{16+18}{2}/12\right)}{33} = .866' \times 12''/' = 10.39'' \rightsquigarrow 11'' \text{ THK SLAB}$$

FOR SHORT SPAN DIRECTION

$W = 1.2D + 1.6L = 1.2(146) + 1.6(40) = 239.2 \rightsquigarrow 240$

$$M_o = \frac{wL^2}{8} = \frac{.240(30)(22' - 28/12)^2}{8} = 349\,Ft\,k$$

$M_u^- = .65(349) = 226.85 \Rightarrow 227\,Ft\,k$

$M_u^+ = .35(349) = 122.15 \quad 122\,Ft\,k$

$b_L = 22'/2 \times 12 = 132$

$b_S = 30/2 \times 12 = 180$

FOR LONG SPAN DIRECTION

$$M_o = \frac{.240(22)(30 - 17/12)^2}{8} = 540\,Ft\,k$$

$M_u^- = .65(540) = 351\,Ft\,k$

$M_u^+ = .35(540) = 189\,Ft\,k$

$A_{s\,min} = .0018(11'')(12'') = .238\,in^2/ft \Rightarrow$

PLACE LONG-DIRECTION CLOSER TO TOP & BOT

ASSUME

$d_L = 11 - .75 - .5/2 = 10''$ (CLR; ½ diam (#4))

$d_S = 10'' - .5 = 9.5''$

$M_n = \frac{M_u}{\phi}$; $\phi = .9$

$N = \frac{A_S}{A_{s\,bar}}$; $A_{s\,bar} = .2$ for #4 bar

$R = \frac{M_n \times 12{,}000}{bd^2}$

$N_{min} = \frac{\text{WIDTH OF STRIP}}{2t}$

MAX SPACING = 18"

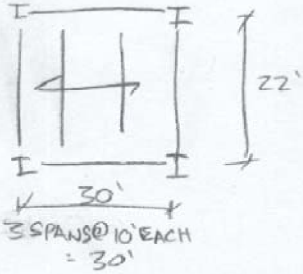
ρ = TABLE A.5

LONG SPAN	STRIP	POSITION	(Ft·k) M_u	(in) b	(in) d	(Ft k) M_n	R	ρ	in^2 A_S	REINFORCEMENT SIZE	N	N_{min}	S	S_m
30'	COLUMN .75 M_u^-	NEG	263	132	10	293	267	.0050	6.6	#4	33	6	4"	18
	.60 M_u^+	POS	113	132	10	126	115	.0020	2.64	#4	14	6	9"	18
	MIDDLE .25 M_u^-	NEG	88	132	10	98	90	.0018	2.38	#4	12	6	11"	18
	.40 M_u^+	POS	76	132	10	85	78	.0015	1.98	#4	10	6	13"	18
SHORT SPAN 22'	COLUMN .75 M_u^-	NEG	170	180	9.5	189	140	.0025	4.28	#4	22	9	8"	18
	.60 M_u^+	POS	73	180	9.5	82	61	.0013	2.22	#4	12	9	15"	18
	MIDDLE .25 M_u^-	NEG	57	180	9.5	64	48	.0010	1.71	#4	9	9	18"	18
	.40 M_u^+	POS	49	180	9.5	55	41	.0010	1.71	#4	9	9	18"	18

Appendix: D

(Composite Deck w/ Non-Composite Steel Framing)

COMPOSITE STEEL DECK W/ NON COMPOSITE BEAMS & GIRDERS (STEEL FRAMING) 1/



$f'_c = 3000$ psi
$f_y = 50$ ksi

LIVE LOAD = 40PSF

DEAD = PARTITIONS = 20PSF
FINISHES & MISC = 5PSF
MEP = 10PSF
35PSF

$1.2D + 1.6L = 1.2(35) + 1.6(40)$
$= 106$ PSF

DECK

USD STEEL DECK MANUAL P. 28

TRY 2" LOK FLOOR 19 GAGE WITH 4 1/2" SLAB
MAX UNSHORED SPAN IS 10.08' > 10' for 3 SPANS. ✓OK
$A_{WWF} = .023\ in^2/ft$ 200 > 106PSF ✓ OK
6X6 W1.4X1.4 WWF = .028 in²/ft > .023. ✓OK

USE 4.5" SLAB w/ 19GAGE 2" LOK FLOOR
W/ 6X6 W1.4X1.4 WWF

BEAM

WHAT IS USED IN USD TABLES.
WT OF CONCRETE = $4.5" - \frac{2}{2} = \frac{3.5"}{12} \times (145\ psf) = 42.29$
DECK WT. = 2.1PSF
S.I.D.L. = 35PSF
APPROX BEAM WT. 75plf

42.29
2.21
+ 35
79.5 ≈ (80PSF)(10') = 800plf + 75plf

DL 875plf
LL = 400plf

$1.2D + 1.6L$
$1.2(.875) + 1.6(.400)$
$W_u = 1.69$ klf

cont'd on next pag

$M_u = \frac{w_u L^2}{8} = \frac{(1.69)(22)^2}{8} = 102.25^{k'}$

DEFLECTION CRITERIA

LIVE LOAD

$\Delta_L \leq \frac{L}{360} = \frac{22 \times 12}{360} = .733''$

$.733 \geq \frac{5 w l^4}{384 EI}$

$.733 \geq \frac{5(.4 kip)(22)^4(1728)}{384(29{,}000) I}$

$I \geq 99.18 \, in^4$

TOT LOAD

$\Delta_T \leq L/240 = \frac{22 \times 12}{240} = 1.1''$

$1.1 \geq \frac{5(1.275)(22)^4(1728)}{384(29{,}000) I}$

1.22E7

$I \geq 210.67 \, in^4$ CONTROLS

W12X30 WORKS ⟹ TRY A BETTER ASSUMPTION FOR WEIGHT OF BEAM.

ASSUME SELF WT = 50 PLF

DL = 830

LL = 400

$1.2(.830) + 1.6(.40) = 1.636 = w_u$ TOT LOAD = .830 + .4 = 1.23

$M_u = \frac{(1.64)(22)^2}{8} = 99.22^{k'}$; $V = \frac{1.64(22')}{2} = 7.04^k$

DEFLECTION

Δ_L = SAME AS PREV. CALCS

$I \geq 99.18$

Δ_{TOT} =

$1.1 \geq \frac{5(1.23)(22)^4(1728)}{384(29{,}000) I}$

$I = 203.23 \, in^4$ CONTROLS.

USE W12X26 $\phi M_n = 140 \geq 99.22^{k'}$ ✓OK $\phi V_c = 84.3 \geq 7.04$ ✓OK

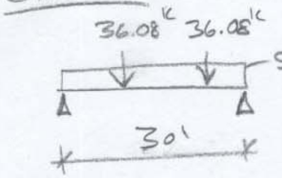
$I = 204 \, in^4 \geq 203.23 \, in$ ✓OK.

* NOTE WE COULD PROBABY GET A W10 TO WORK BUT IT IS NOT AS ECONOMICAL. THOUGH THERE IS A HEIGHT LIMITATION A W10 & W12 DONT MAKE A BIG DIFFERENCE. 15 stories x 2" = 30" DIFFERENCE

(Cont'd on next pag.

3

GIRDER

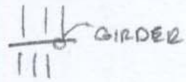


PT LOADS

↳ $w_u = (1.64 k/ft)(22') = \frac{36.08'}{2} = 18.04^k \times 2 = 36.08$

FOR 2 BEAMS JOINING

*GIRDER IS BEING DESIGNED FOR CONTINUOUS BAYS



$\Delta_{TOT} = \frac{Pl^3}{28EI} + \frac{5wl^4}{384EI}$ ASSUME 80lb/ft S.W. OF GIRDER

$\Delta_T = \frac{(36.08)(30^3)(1728)}{28(29{,}000)I} + \frac{5(.08)(30)^4(1728)}{384(29{,}000)I}$ $L/240 = \frac{30 \times 12}{240} = 1.5''$

$1.5 = \frac{2073.09}{I} + \frac{50.2759}{I}$

$1.5 = \frac{2123.37}{I}$

$I \geq 1415.58$ CONTROLS.

NEED TO MAKE A BETTER SW. ASSUMPTION, A LOT MORE THAN WHAT I THOUGHT IT WAS.

ASSUME 130lb/ft SW. GIRDER

$1.5 = \frac{\left(2073.09 + \frac{5(.130)(30)^4(1728)}{384(29{,}000)}\right)}{I}$ 81.698/I

$1.5 = \frac{2154.79}{I} \Rightarrow I = 1436.53 in^4$

TRY W12X170: WE NEED TO STAY WITHIN RANGE OF 12"-14" FOR THE BEAM OR WE WILL LOSE TO MANY FLOORS.

$M_u = 36.08(10) + \frac{.170(30)^2}{8} = 379.93^{k'}$

redo deflection CALCULATION

$1.5 = \frac{2073.09}{I} + \frac{5(.170)(30)^4(1728)}{384(29{,}000)/I} \Rightarrow$ $1.5 = \frac{2177.93}{I}$

$I = 1453.28 in^4$

Cont'd on next page

USING W12X170

$\phi M_n = 1030 \geq 379.93$ ✓OK

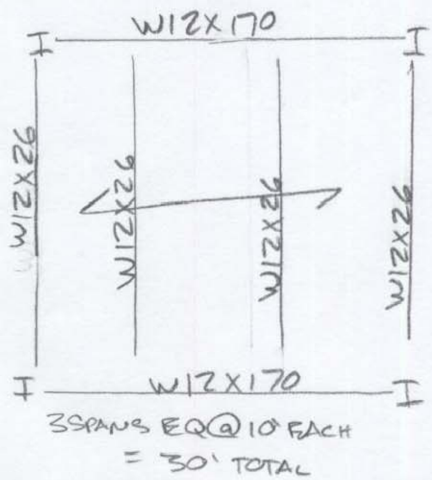
$I = 1650 in^4 \geq 1453.28 in^4$ ✓OK

$V_u = 36.08 + \frac{(.170)(30)}{2}$

$V_u = 38.63$

$\phi V_n = 404 \geq 38.63$ ✓OK

* DEFLECTION IS SIGNIFICANTLY CONTROLLING THIS DESIGN. KEEP NOTED THAT I WANT TO KEEP THE FLOOR SANDWICH AS THIN AS POSSIBLE & W12X170 WAS THE SMALLEST SIZE THAT WOULD WORK FOR I REQ



DECK 4.5" SLAB W/ 19 GAGE 2" LOK FLOOR W/ 6X6 W1.4XW1.4 WWF

BEAMS - W12X26

GIRDERS - W12X170

TOTAL SLAB DEPTH = 14" (W12X170) + 3.5 (SLAB + DECK) = 17.5" DEPTH

TYP. INT BAY